ENGLISH VERSION

Overview of the Structural Behavior of Columns, Beams, Floor Slabs and Buildings during the Earthquake of 2016 in Ecuador

Una mirada al comportamiento estructural de columnas, vigas, entrepisos y edificaciones durante el sismo de Ecuador 2016

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Abstract

This paper analyzes different types of failures in buildings of the cities of Portoviejo, Manta and Bahía de Caráquez during the earthquake of April 16, 2016. The evidence derived from the failures refer to existing codes and design considerations in order to reveal how modeling corresponds or not with different reinforced-concrete columns, floor slabs and rigid frame buildings without structural walls. Given the diversity of constructive solutions found in the steel-deck slabs, a 2 factorial experiment design was performed to identify the influence and independence of the type of node, the rectangularity of the floor plan and the height of buildings on the strength and deformation parameters of floor slabs, in relation to the failures observed. Additionally, this paper undertakes a singular analysis approach of different failure causes in a building demolished in the city of Portoviejo, based on evidence revealed by the earthquake, and described some of the distinctive features of this type of study.

Keywords: *Failures, buildings, earthquake, factorial design of experiments, steel-deck*

Resumen

En el trabajo se hace un análisis de diferentes tipos de fallas presentadas en edificaciones de las ciudades de Portoviejo, Manta y Bahía de Caráquez durante el sismo del 16 de abril de 2016. Las evidencias proporcionadas por las fallas se relacionan con códigos vigentes y consideraciones de diseño para revelar cómo la modelación se corresponde o no con lo construido en diferentes casos de columnas, entrepisos y edificios porticados de hormigón armado sin muros estructurales. Dada la diversidad de soluciones constructivas encontradas en los entrepisos tipo steel-deck se realizó un diseño de experimentos factorial 2^3 para identificar la influencia e independencia del tipo de nudo, la rectangularidad en planta y la altura de los edificios en los parámetros resistente y deformacionales de estos entrepisos, asociados a las fallas observadas. Además se realiza un acercamiento al análisis singular de la falla multicausal de un edificio demolido en la ciudad de Portoviejo a partir de las evidencias reveladas por el sismo y se describen algunas de las particularidades de este tipo de estudio.

Palabras clave: Fallas, edificaciones, sismo, diseño factorial de experimentos, steel-deck

1. Introduction

On April 16, 2016, Ecuador suffered an earthquake with a moment magnitude (Mw) of 7.8, whose epicenter was located at a depth of 20 km in the coast of Pedernales. It was caused by a plate subduction of the same origin as the earthquakes of January 1906 (Mw 8.8), May 1942 (Mw 7.8), January 1958 (Mw 7.8), December 1979 (Mw 8.1) and August 1998 (Mw 7.1). The cities of Pedernales, Portoviejo, Manta and Bahía de Caráquez of the province of Manabí were most affected, with measured accelerations ranging from 1.407g PGA(E) (Pedernales) to 0.38g PGA(N) (Portoviejo). Additionally, an acceleration spectrum obtained in the Portoviejo station exceeded the elastic spectrum of the prevailing Ecuadorian Construction Standard (NEC). Although this station is located on a Type D soil (IGEPN, 2016), which is better than the most damaged area, having a Type E and F soils (NEC, 2015a) that were affected by floods of the late rainy season and the El Niño phenomenon.

At an undesirable and priceless cost for any society, the earthquake turned these cities into a real-scale laboratory for the civil engineering field. The investigations dealing with the affected constructions ranged from simple observation and analysis to measurements, modeling or reverse engineering for identifying damages.

Damages always teach lessons that, once recorded and analyzed, allow a better understanding of engineering, and prevent future failures (Gallegos, 2011). The phrase "Failures: masters of engineering" (Gallegos, 1998) warns about the evidences of real stress-strain conditions of the works, contributed by failures, and their possible contradictions between the real conception and the physical and mathematical models used in their design.

A famous structural engineer referred that he had never calculated a structure… only structural models. Because the art of engineering, he said, consists in developing designs and adopting models that fit the actual behavior that works will show during their entire service life; thereby recognizing that structures would never care about which model the designer had developed if their behavior were

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different and a failure should occur. For example, if an engineer designs a self-supporting, cantilever, reinforced-concrete slab, supporting a rectangular spandrel of the same material, monolithically, he will discover that the spandrel will crack at the time of stripping. Because the slab deformation and the principle indicating that the load will search the element giving it the greatest stiffness, geometrically or mechanically, even if it was not designed that way in the model.

In the words of Roberto Meli Piralla, "Structural design is the art of using materials that we actually don't know, in order to form structures that we actually can't analyze, so that they can resist loads that we actually can't evaluate, in such a way that people will not notice our ignorance" (Meli, 2001:71). Therefore, failure analyses during an earthquake are not only a right but also a duty of contemporary engineering.

This paper presents and systematizes structural failures in columns, steel-deck slabs, rigid frames without structural walls, and high building spandrels in the cities mentioned above; additionally, it analyzes their possible causes.

Failures should be investigated from different methodological perspectives (qualitative, hypothetical-deductive, etc.) in order to learn from them. This work combines these perspectives in order to analyze column failures, based on standards of the American Concrete Institute (ACI-318-95, 1995), (ACI-318S-05, 2005), (ACI-318S-14, 2014) and their adaptations in the 1993 Ecuadorian Construction Code (CEC) and the Ecuadorian Construction Standard (NEC) of 2011 and 2015. The authors of the paper took the photographs, which were subjected to a previous classification and selection. The satellite image is from Google Maps.

A qualitative study of failures and a 2^3 factorial experiment design were performed in steel-deck slabs, with steel-concrete composite grids, with the aim of analyzing the influence of the building's plan and height and the "hybrid node" type (Mieles and Castañeda, 2016b) on their stress-strain indicators (moment, shear and floor drift) for gravity and seismic load combinations. Finally, evidence was gathered and preliminary assessments were made regarding the failure of a significant building in Portoviejo, demolished by implosion on August 7, 2016.

2. Column failure

Column failures are usual during an earthquake, and it is an frequent cause of collapse in buildings. The investigated cities gradually modified their columns, introduced new materials and construction technologies, and transformed their regular columns, with similar mass distribution and limited slenderness in low buildings (one or two floors), whose greatest risks were the tapping with adjacent buildings. The modifications consisted in thin and slender columns, irregularly distributed or omitted in the floor plan, with different mass distribution, plus cantilevers and eccentric loading, with a more modern and functional architecture, which demands a greater high-standard engineering and constructive culture, in a high-seismicity region not suitable for these types of construction (Figure 1).

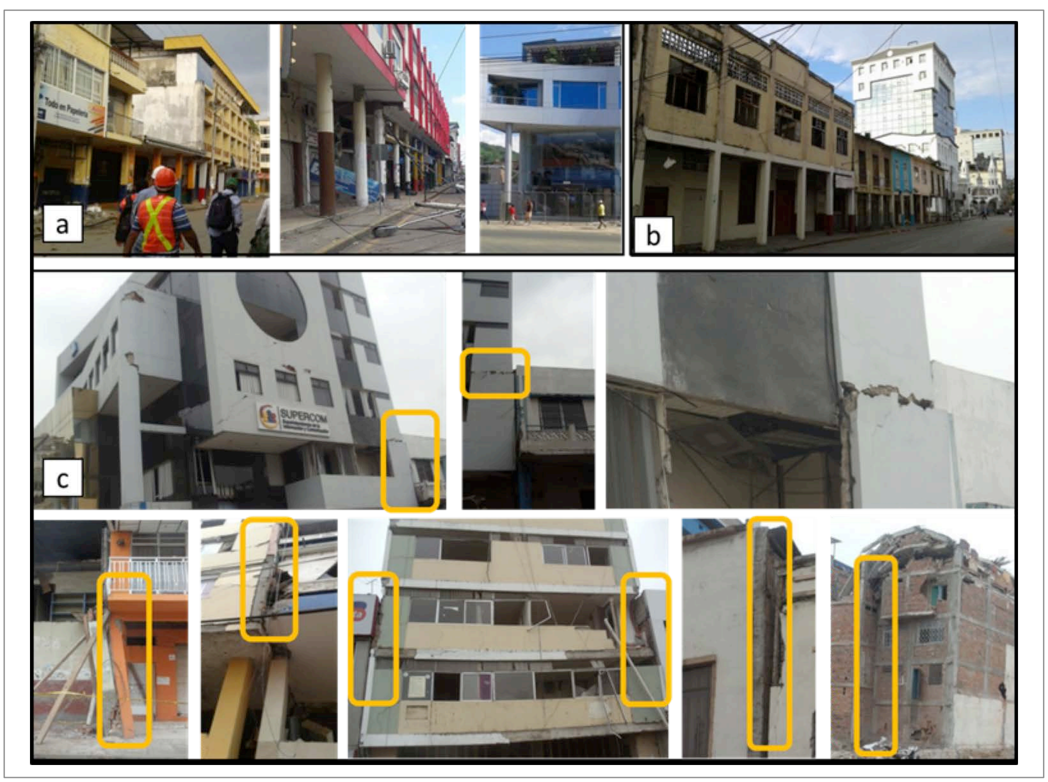


Figure 1. *a) Changes in column slenderness due to changes in the urban architecture; b) areas with continuous changes in the architecture of the city; c) Pounding damage from adjacent buildings, built in the property limit, in different periods and with diverse typologies*

Thus, some buildings, which did not collapse by acceleration fractions, show confinement zones that concentrated shear failures in the column ends (Figure 2); within the expected limits for l_0 in heading 21.4.4.4 of CEC 1993 and L_0 in section a of heading 4.3.4 of NEC SE-HM 2011 and 2015, based on ACI 318S-14, 2014.

Moreover, in many columns, the stirrups' spacing did not comply with 21.4.4.2 of CEC 1993 nor with section b of 4.3.4 of NEC SE-HM 2011 and 2015. The standard requires a maximum spacing of 70 mm, for 12 mm steel columns or smallest side of 300 mm. Furthermore, the lack of confinement and seismic hooks, due to constructive negligence or non-application of the standard, were the probable cause of several failures, in addition to an insufficient use of spiral reinforcement and enclosure hanging beams that would allow better distributing the seismic drag force, in its interaction with the slabs (Figure 3).

Additionally, the following column failures were observed: new floor slabs were built over low buildings, increasing their center of mass and creating "soft floors", thereby failing to comply with 21.4.4.5 of CEC 1993 and 4.3.4.d of NEC 2011 and 2015, because columns were not continued and stirrups were not placed in the mid-span of the original spacing. Furthermore, short columns were added (Aguiar, 2010), and columns were designed without a bracing system, thus modifying their dynamic properties while braced to partition walls (López and Espinoza, 2016). Moreover, weak columns with strong beams, together with columns of less than 300 mm and poor steel detail, which did not comply with the established standards either, thereby causing brittle failures (Aguiar, 2008; Blanco, 2012, ACI-318S-14, 2014), were observed (Figures 3 and 4).

In these cities, another failure cause was the lack of any reinforced-concrete corner columns in the ground floor, for allegedly architectonical reasons, but inacceptable from the engineering point of view in such high-seismicity regions (Figures 5 and 6).

Figure 6 shows several collapsed buildings without corner column in the ground floor. Witnesses of the 6-b building confirmed that they saw its diagonal swing before falling down, a fact that can also be deduced from the photos of the 6-a building. The remaining 6c-6f suffered shear failure and soft floors.

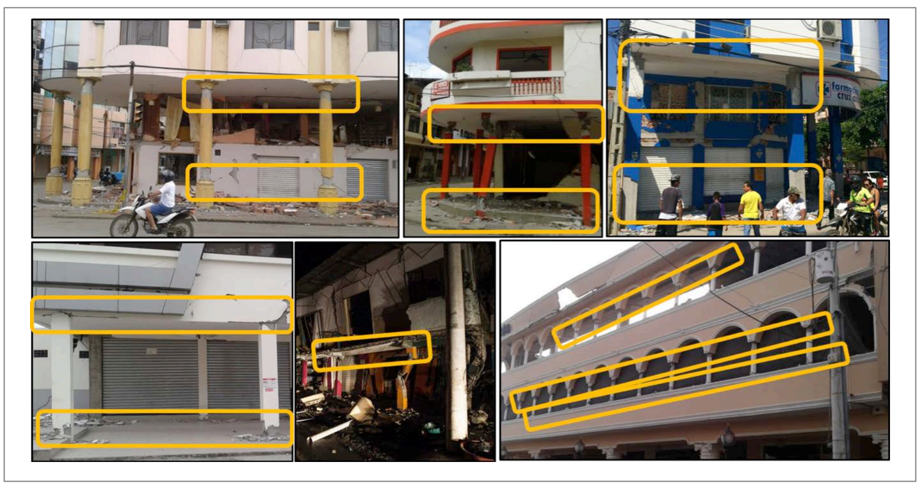


***Figure 2.** Building of the city of Portoviejo showing L_0 zones where shear failure and column confinement are concentrated*

***Figure 3.** Seismic-related damage was amplified by technological indiscipline, violations of code regulations, and negligence in construction*

Figure 4. *Low buildings collapsed by column failures*

Figure 5. *Collapsed building in Portoviejo, without corner column in ground floor*

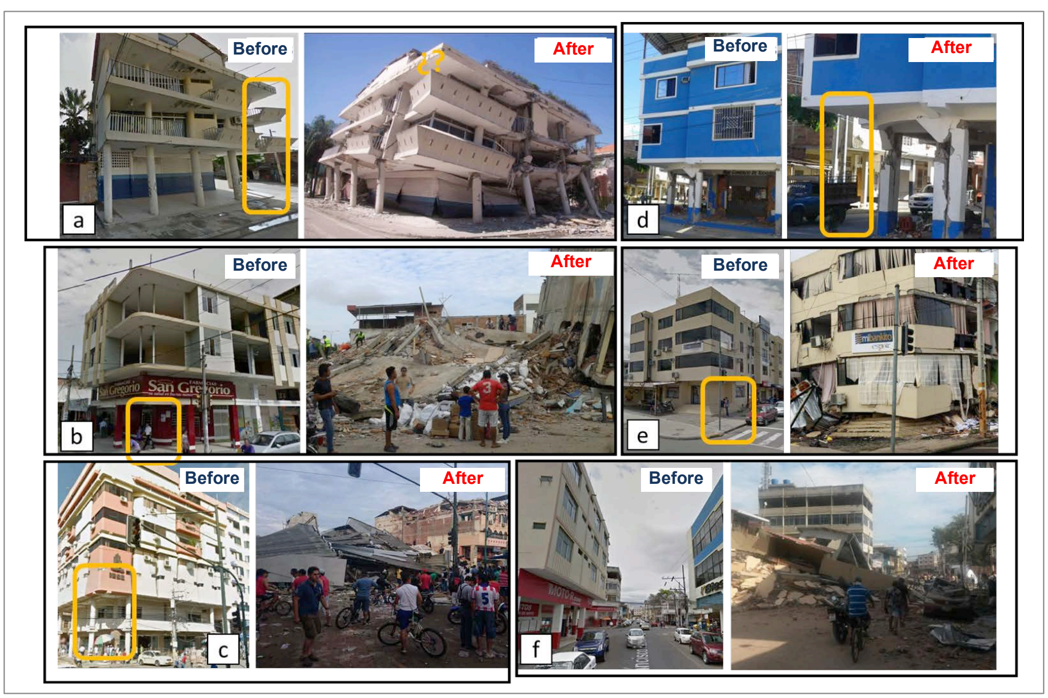


Figure 6. *Other collapsed buildings in Portoviejo, without corner column*

3. Failures in steel-deck slabs

Composite grids of reinforced-concrete primary beams with or without steel secondary beams, totally or partially integrated with composite slabs with galvanized steel decking for supporting floor slabs and roofs, are widely used in Ecuador, motivated by suppliers who have established technical-constructive conditions and promoted their technical-economic advantages. These solutions have reduced the use of formwork wood, the environmental impact, and the execution schedules and costs, in relation to the traditional system of lightweight slabs and wood formwork, by a 20% range of the total cost. They have also met municipal regulations regarding cleanliness, occupation of public spaces and order during works (Plasencia et al., 2014), creating (concrete-steel) connection points, called "hybrid nodes" (Mieles and Castañeda, 2016b), with multiple constructive alternatives and structural behaviors, which are still not sufficiently structured and correlated among them.

A preliminary damage analysis of ten buildings built with this technology showed a good structural behavior of steel-deck slabs with these grids, which distributed the lateral load and the floor drift among the vertical elements (Mieles and Castañeda, 2016c). However, given the numerous constructive alternatives, with steel beams in one or two directions, columns in the vortices, and frames closed by non-structural masonry walls or not (Figure 7), an experimental design was required to determine the influence of the height of the building, the rectangularity of the plan, and the physical-mechanical model of the nodes under gravity and seismic loads, on the stress-strain indicators of the slabs. This allowed correlating or not the damages, with just the specific characteristics of these grids.

These buildings showed no slab swelling caused by loss of stability in their plan, and the loss of local stability in steel beams and steel decks was reduced (Figure 8).

There were multiple evidences of displacement or rotation of the steel beams in the "hybrid nodes" with the reinforced concrete beams (dowel effect) (Figure 9), which showed very small damages and good energy dissipation during the earthquake (Perea, Mora and León, 2014).

Figure 7. *Steel-deck slabs in the cities of Manta, Portoviejo and Bahía de Caráquez*

Figure 8. *Local failure by buckling of steel decks and steel beams*

Figure 9. *Evidence of displacements of steel beams in the hybrid nodes*

Buildings in progress (without their whole gravity load) and with severe constructive irregularities, did not collapse (Figure 10). They just showed buckling and deflection of some steel beams, which are easily replaced by shear and welding.

Figure 11 shows the cracking patterns in the hybrid nodes of primary beams. They initiate at the contact interface between one steel beam face and the bonded concrete, and they spread 45° towards the tension zone, or perpendicular to the steel beam. Moreover, cracks appear in the bended side and center of the spans, from the neutral axis to the lower part of the concrete beam at an angle of 45° or 90°, while in areas close to the support, shear cracks are extended from the neutral fiber to the upper edge of the concrete beam (Mieles and Castañeda, 2016c).

These patterns confirm the dowel effect through hybrid nodes (Figure 9 and 11) and the presence of geometric and load discontinuities with stress concentrations in both elements, which create a D region (ACI-318S-14, 2014; Schlaich, Schäfer and Jennewein, 1987). Where the hypothesis of flat sections does not apply to the design, because in this region, the principles of compatible deformation nor the corresponding force balance, are not satisfied (ACI-318S-14, 2014). Modeling of these nodes by the Finite Element Method (FEM) shows a qualitative change in the isostatic network of the trajectories of internal stresses in the hybrid node of the concrete beam (Mieles and Castañeda, 2016a), which turns the issue of spatial layout and physical-mechanical behavior model of the node into a pertinent and current research subject.

Other researches dealing with hybrid nodes in steel columns and concrete beams (Bai, Nie and Cai, 2008; Rasoul, Bakhshayesh and Mehdi, 2016), steel columns and concrete slabs (Aznar, García, Herrera and Cervera, 2008), and shear walls with steel beams (Soto, 2012) differ from these "hybrid nodes" (Mieles and Castañeda, 2016b), because steel beams are not ducts and the concrete's shear strength cannot be calculated with the equation of 22.5.5.1 of ACI 318S-14, because the nodes interrupt the concrete's mechanical bond.

Furthermore, the earthquake revealed that fasteners between the steel deck and the steel secondary beams were few and weak, which limited their joint work and caused the steel beam grid to be somewhat disconnected from the steel-deck slab.

Figure 12 defines three independent variables used in the 2^3 experimental design to evaluate its influence on the strength behavior of "hybrid nodes" and the floor drift in rigid frame buildings without structural walls, secondary steel beams in one direction, and steel-deck slabs.

Five independent variables were selected: Bending moments (M) and shear forces (V) in hybrid end nodes (HEN) and hybrid intermediate nodes (HIN) of steel beams as strength variables, and the inelastic floor drift (IFD) as a strain variable. In each case, the 95^{th} percentile was calculated from the dependent variable with low coefficients of variation (CV) for gravity and seismic loads that justified their homogeneity and representativeness for the regression. Calculations assumed load combinations of ACI 318-14 (5.3.1.b for gravity loads and 5.3.1.e for seismic loads). The numerical experiments were calculated with the ETABS software (Computers and Structures, 2015; Guerra, 2015; Mieles and Macías, 2015). As an example, the self-prepared Table 1 shows the coefficients of variation (CV) and the values obtained from the six dependent variables and the eight load combinations in 5.3.1.e.

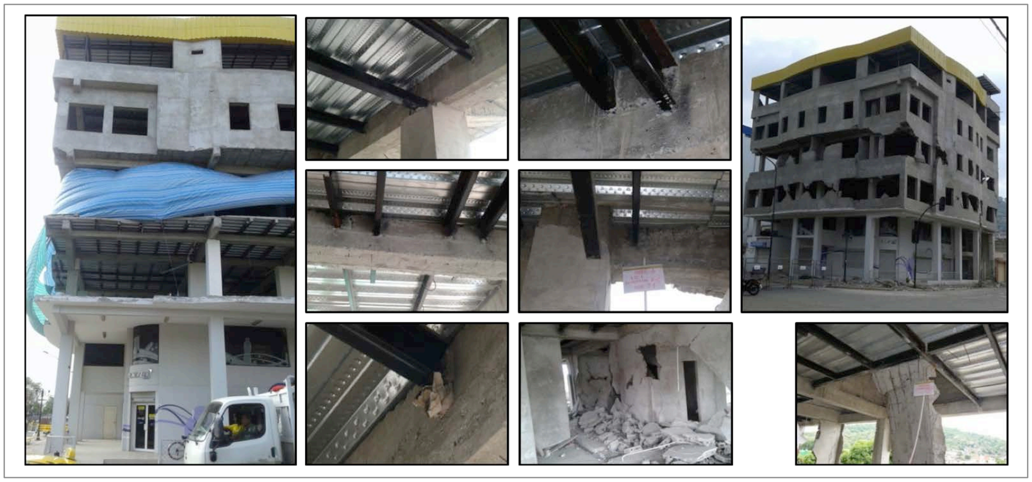


Figure 10. *Building in progress, with constructive irregularities, not collapsed during earthquake*

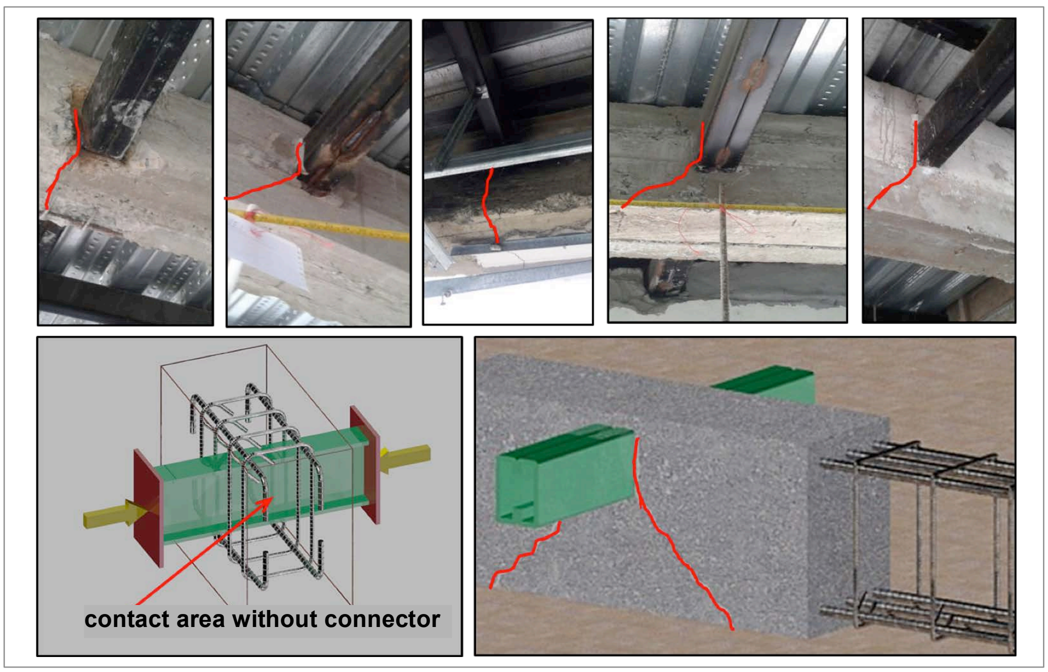


Figure 11. *Checking and crack patterns observed in the nodes area*

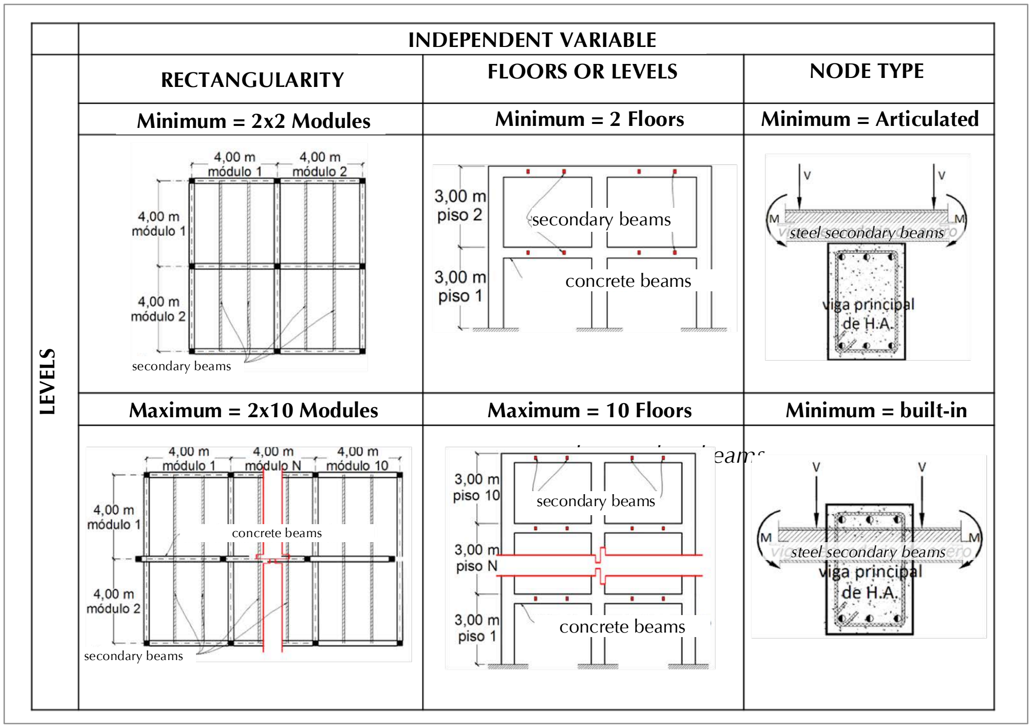


Figure 12. *Types and levels of independent variables used in the experimental design*

Table 1. *Results of Dependent Variables ($P_{0.95}$) for Load Combinations 5.3.1.e in the Experimental Design*

Cases	Independent Variables (IV)						Dependent Variables (DV)									
	Levels of IV						Combination for 5.3.1.e (1,2 D + 1 Earthquake + 1 L)									
	Niveles		Rect.		Tipo de Nudo		Mf NHE (kN-m)		V NHE (kN)		Mf NHI (kN-m)		V NHI (kN)		DPI (10-3 m)	
							CV	M ($P_{0.95}$)	CV	V ($P_{0.95}$)	CV	M ($P_{0.95}$)	CV	V ($P_{0.95}$)	CV	Δ_I ($P_{0.95}$)
1	-	2A	-	B 1:1	-	C1	0	0	0,002	14,00	0,007	18,84	0,001	23,10	0,095	3,59
2	+	10A	-	B 1:1	-	C1	0	0	0,007	14,32	0,021	20,06	0,004	23,41	0,331	7,73
3	-	2A	+	B 1:5	-	C1	0	0	0,002	14,04	0,008	19,02	0,002	23,15	0,097	4,59
4	+	10A	+	B 1:5	-	C1	0	0	0,008	14,40	0,023	20,34	0,005	23,47	0,336	9,27
5	-	2A	-	B 1:1	+	C2	0,024	12,67	0,007	18,70	0,015	13,90	0,005	19,40	0,091	3,52
6	+	10A	-	B 1:1	+	C2	0,062	15,87	0,022	20,21	0,049	15,96	0,019	20,34	0,330	7,63
7	-	2A	+	B 1:5	+	C2	0,028	12,98	0,009	18,85	0,017	14,24	0,006	19,58	0,089	4,45
8	+	10A	+	B 1:5		C2	0,068	16,48	0,025	20,51	0,054	16,50	0,021	20,62	0,334	9,11

The experimental design determined the influence of the independent variables and the simple (SI) and double (DI) interaction among them on the values of dependent variables (Pulido and De la Vara Salazar, 2012), by means of two hypothesis:

> *H_0: The independent variables or the interaction of these variables* ***do not influence*** *the value of dependent variables.*
>
> *H_1: The independent variables or the interaction of these variables* ***influence*** *the value of dependent variables.*

The result obtained a p-value that, for values less than 0.05, demonstrates that there is enough statistical evidence so as to reject H_0 with a 5% significance level. The results for each dependent variable and load combination were obtained by a standard statistical application (Statgraphics Centurion, 2009). As an example, the self-prepared Table 2 summarizes the p-values for the experimental design regarding the combination 5.3.1.e in the expected interactions, which show the majoritarian independence of the steel grid strength variables in relation to the building's height and rectangularity, thus justifying the previous qualitative analysis.

Dependent variables (M_{fk} and V_k) showed a maximum CV of 0.06 and a statistically significant influence in relation to the physical-mechanical model of the node and a partial influence of the building height in simple and double interactions.

The last columns of Table 2 confirm that the IFD is the less homogenous dependent variable, with CV less than 0.34, and that 3.0 m supports cause an elastic displacement between 3.6 mm and 9.2 mm and an inelastic displacement to 0.75 R of those values; according to NEC (Ecuadorian Construction Standard), these values should not mobilize an unacceptable stress-strain response in non-structural walls.

However, failures in RC rigid frame buildings with steel-deck slabs were often caused by the absence of structural walls for reducing the flexibility of the frames and the floor drift. This caused non-structural walls (spandrels and partition walls) to suffer unacceptable displacements and massive collapse under horizontal load, which, according to the modeling, should have been supported by the columns (Figure 13).

Table 2. *Results of the Regression of the Experimental Design for the Load Combination 5.3.1.e*

VI	SECONDARY BEAMS IN ONE DIRECTION Gravity Loads + Earthquake (Combination 1,2 D + 1 EARTHQUAKE + 1 L)														
	M (NHE)			V (NHE)			M (NHI)			V (NHI)			DPI		
Simple Interaction (SI)	Estim.	T	P-Value	Estim.	T	P-Value	Estim.	T	P-Value	Estim.	T	P-Value	Estim.	T	P-Value
A. Height	0,838	1,978	0,029	0,481	3,054	0,038	0,116	7,392	0,002	0,326	3,724	0,020	22.00	31,885	0,0000
B. Rectang.	0,115	0,272	0,201	0,071	0,452	0,675	0,116	1,444	0,222	0,071	0,813	0,462	6,193	8,97533	0,0009
C.Node Type	7,250	17,12	0,003	2,689	17,06	0,000	0,116	19,03	0,000	1,649	18,82	0,000	0,578	0,83674	0,4498
Constants	7,250	17,12	0,000	16,88	107,1	0,000	0,116	149,6	0,000	21,63	246,9	0,000	62,38	90,4064	0
Variance Analysis Double Interaction (DI).	Σxi2	F	Valor P	Σxi2	F	Valor P	Σxi2	F	Valor P	Σxi2	F	Valor P	Σxi2	F	Valor-P
A. Height	1,675	498,8	0,029	1,853	1225,	0,018	1,715	4706,	0,009	0,852	841,0	0,022	3871,	5506442	0,0000
B. Rectang.	0,230	9,400	0,201	0,041	26,85	0,121	0,335	179,6	0,047	0,041	40,11	0,100	306,8	4363134	0,0001
C.Node Type	14,50	3738	0,003	57,84	38238	0,003	4,415	3119	0,004	21,75	2148	0,004	2,666	379210	0,0010
AB	0,075	1,000	0,500	0,005	2,980	0,334	0,075	9,000	0,205	0,002	1,490	0,437	15,02	213569,	0,0004
AC	1,675	498,8	0,029	0,775	512,4	0,028	0,445	316,8	0,036	0,228	225,0	0,042	0,025	3576,0	0,0106
BC	0,230	9,400	0,201	0,014	9,000	0,205	0,105	17,64	0,149	0,015	15,12	0,160	0,191	27225,	0,0039

Figure 13. *Rigid frames and steel deck slabs without structural walls*

In order to confirm this, the maximum floor drift ($P_{0.95}$) was calculated for 5.3.1.b and 5.3.1.e in the extreme cases of Table 1, including two structural walls in both directions. The resulting maximum drifts of 0.00088 and 0.00196 provoked maximum displacements of 2.63 mm and 5.88 mm, which did not cause the collapse of non-structural walls (Figure 14), and lower than 4.5 mm and 9.2 mm in rigid frames without structural walls.

A number of rigid frame buildings without structural walls had block rectangular boxes for service facilities (electrical, hydraulic, etc.) close to other partition walls, thus creating areas with high geometrical stiffness, without physical-mechanical stiffness to support horizontal loads. Therefore, when the IFD occurred, they attracted load for which they were not designed, thus increasing the deformation damage in the building without affecting the rigid frame (Figure 15-b). Figures 15-a and 15-c show other cases of double walls that attracted horizontal loads and caused the failure.

The excess of plaster (6 to 9.5 cm) caused other failures in buildings with steel-deck slabs and rigid frames without structural walls, which created sandwich walls of 30 cm thick that attracted seismic loads that were not considered in their physical-mechanical design (Figure 16-a).

A plaster sample showed increases of 112 kg/m^2 per face, transforming the block wall into a wall of 300 kg/m^2, sometimes with no continuity in the ground floor. This caused beam failure, because the load was triplicated and the mass hanging from the cantilevers increased, thus evidencing shear in upper floors that they could not resist (Figure 16-b), and affecting the vibration period of the building (López and Espinoza, 2016) (Milheiro, Rodrigues and Arêde, 2016) (Priestley and Paulay, 1992).

A questionable constructive and structural conception of the stairs located them between non-structural block walls that collapsed together with the stairs (Figure 17-a); or were simply supported on floor slabs (Figure 17-b) and were not protected between structural walls that would form strong nucleuses that would resist horizontal loads.

4. Preliminary approach to failures in a symbolic building of this earthquake

In the same way that the Hanshin highway required a detailed investigation after the Kobe earthquake, the structural behavior of a building requires studies that exceed the purpose of this work. The building of the Pichincha Mutual Company in the city of Portoviejo, with reinforced-concrete columns and beams, and steel-deck slab without steel secondary beams, was severely damaged during this earthquake. It was demolished by implosion on August 7, 2016, and ten days after, debris were removed from the place, taking along everything that engineering could have learnt from them (Figure 18). Figure 18.a, taken from Google Maps, shows the building before the earthquake; Figure 18.b, its consequences; and 18.c, its demolition and evacuation.

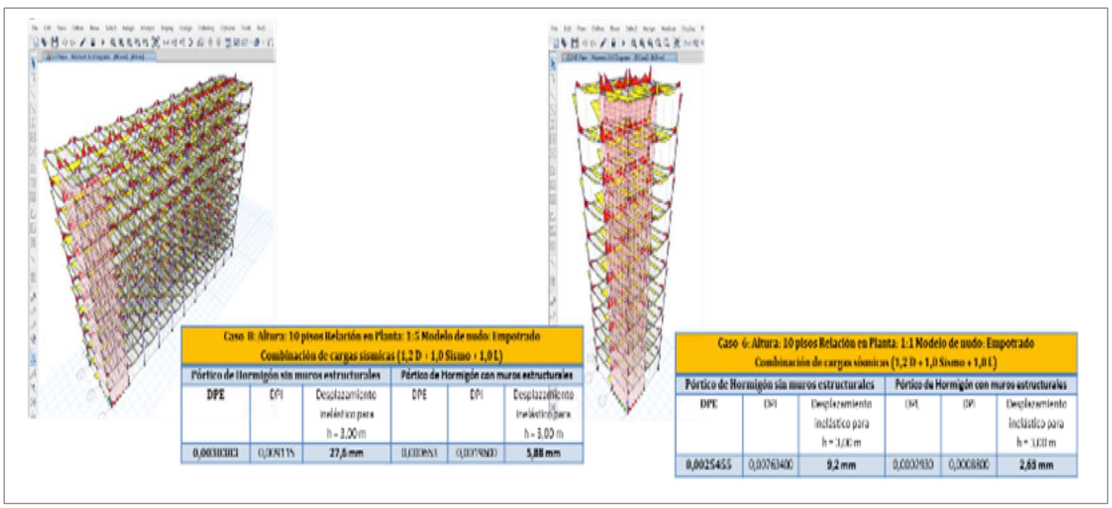


Figure 14. *Models and results with ETABS of extreme cases for comparing IFD*

Figure 15. *Duplication of walls and vertical ducts with high geometrical stiffness create false diaphragms that attract horizontal loads without having the physical-mechanical capacity to support them*

Figure 16. *High plaster thickness generated sandwich walls and attracted seismic load that they could not resist*

Figure 17. *Inadequate conception of the stairs anchorage and their location in non-structural "boxes"*

Figure 18. *Beginning and end of a building damaged by the earthquake of April 16, 2016, which took along many lessons that could have been useful to the engineering field*

A presumptive diagnosis, a research plan, core sample extraction, the development and negation of hypotheses and other aspects, including legal ones, should be systematically considered in this kind of investigation. However, in this "overview", these actions had only a non-systemic character that did not surpass the most speculative diagnoses (Figure 19).

The Pichincha Mutual Company, with northeast-southwest facade, was constituted of ten levels: ground floor, a mezzanine and eight floors, each one on top of a reinforced-concrete beam grid with a span ratio over 1:8, sustaining steel-deck slabs, supported by circular and rectangular columns of different slenderness in the ground floor. The whole building was built over a concrete slab and three out of four faces were cantilevered (Figure 20).

One look at the building allows identifying short columns; weak columns together with string beams (NEC-SE-RE, 2015); and high beams with l/d=4 ratios associated to a possible shear failure (NEC-SE-HM, 2015) (Figure 21).

The accelerograms measured during the earthquake and the orientation of the building demonstrate that the failure and directionality of the earthquake coincided with their floor plan diagonal (North-South). A preliminary dynamic analysis recognizes the failure in the second oscillating mode and the interrelations between peaks of the accelerograms for the first 15 seconds of the earthquake, and the recovery period of the normal oscillating mode of the building, as one of its possible failure causes (Figure 22).

Figure 19. *The investigation of a building needs measuring, taking core samples, and even discovering and recognizing that, sometimes while investigating, you may lose the answers*

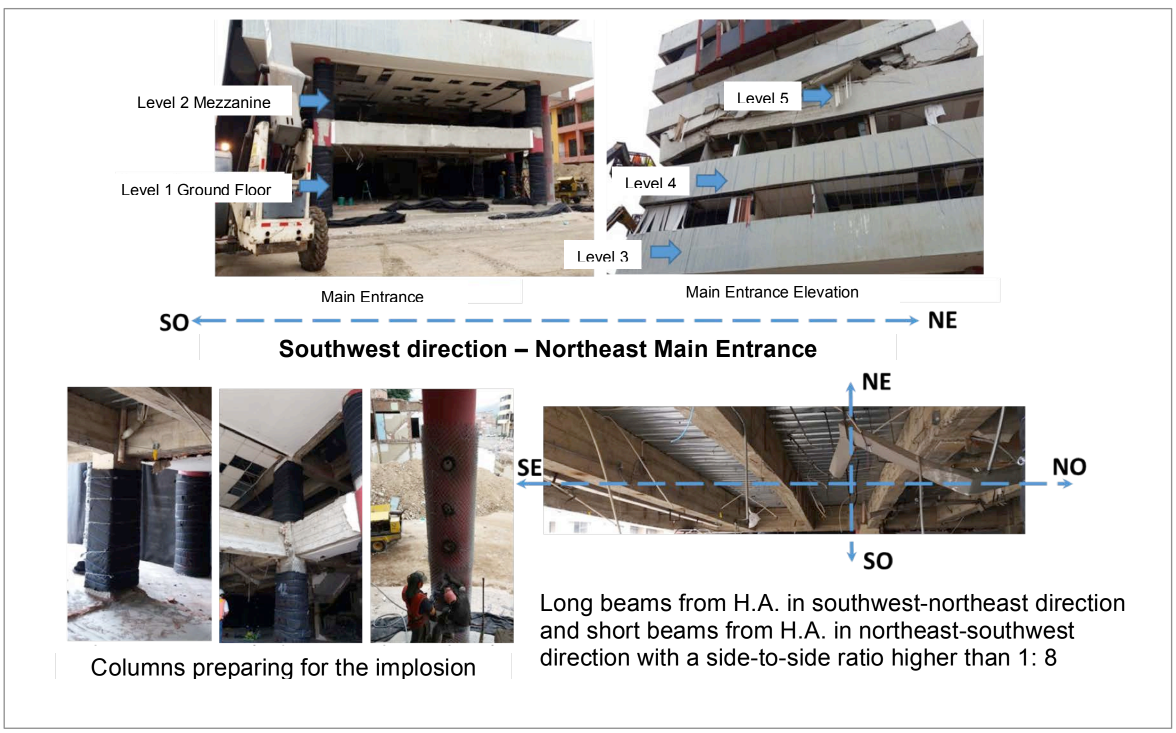
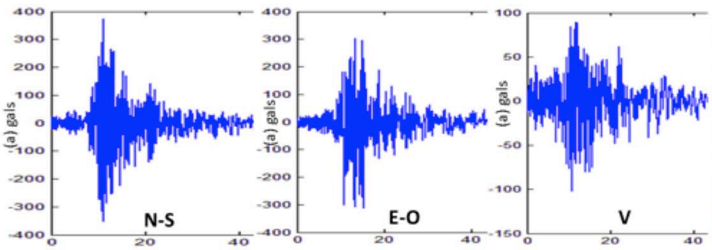
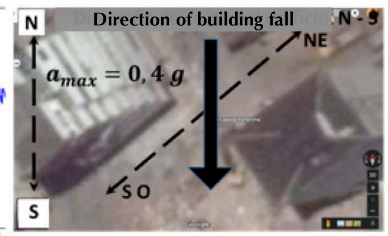


Figure 20. *General elements of the building characterization*

Figure 21. *Short columns and weak columns facing strong beams*



Accelerogram for the first 40 seconds of the earthquake in Portoviejo. (Aguiar, 2016)



Main facade address of Mutualista Pichincha. NE – SO (Google Maps)

Figure 22. *Earthquake-building reference orientation for a preliminary dynamic analysis*

The Technical University of Manabí revealed images of the 4th and 5th level of the building, where columns resisted the fall of the upper floors without an immediate collapse. They also show shear failures in the RC short columns and the effect on the longitudinal long beams, thus turning the plan of upper floors into the surface of a hyperbolic paraboloid over two crossed straight lines. This is consistent with the hypothesis of horizontal and vertical shear in the floor slabs of the 6th and 7th level during the earthquake (Figure 23).

Additionally, the floors' impact on the columns caused a stability loss of longitudinal reinforcing steels and horizontal tensile strength by Poisson effect, which opened stirrups and revealed irregularities during the building construction. For example, overlapping of longitudinal bars in the same section, too much spacing between stirrups, and absence of confinement nucleus in areas with large amount of longitudinal elements, as standards require (paragraph 10.4 of ACI 314R-11) (Figure 24).

The influence of the ducts' size and position in RC I-beams as a failure hypothesis, concealed ever since the construction of the building, is an issue that revives with the photos revealing multiple inclined cracks that cross or are born in ducts located at places not complying with the standards. There are even traces of repairs before the earthquake, which may be related to the overall failure of the building and will remain within the hypotheses that this "overview" was unable to find out (Figure 25).

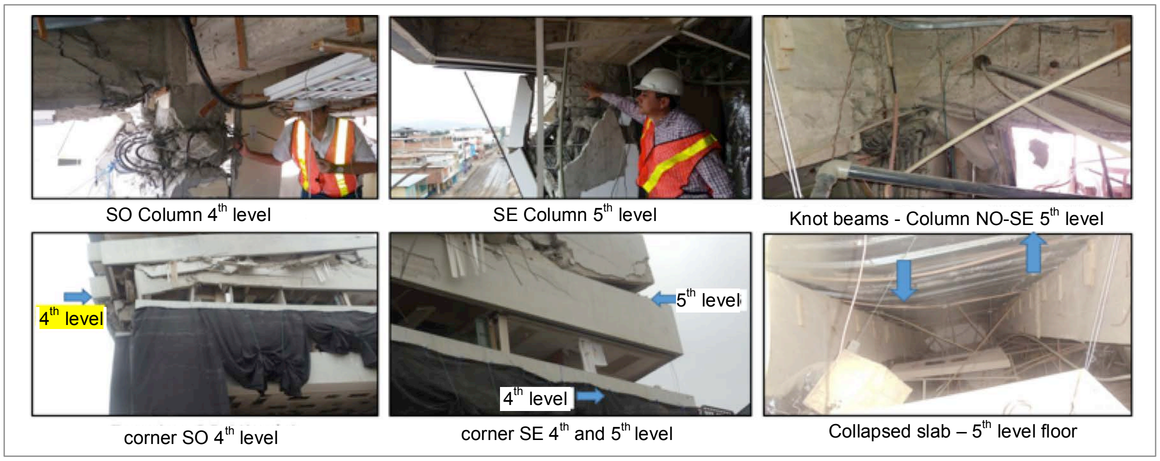


Figure 23. *Columns and beams of the 4^{th} and 5^{th} level*

Figure 24. *Irregularities revealed by the earthquake in the construction of the building*

Figure 25. *Inclined cracks crossing the ducts in I-beams*

5. Final considerations

Pathologists rely on protocols for examining a body until they can determine the cause of death or possible epidemic disease. During these protocols, several specialists working at the morgue and specific laboratories (microbiology, parasitology, etc.) use standardized tests to process data and contribute with the necessary information to synthetize a pathologic diagnosis that allows fighting diseases and improve the quality of life of living beings. A pathological research also includes an anamnesis studying the background of the context in which the dead person lived as well as the person's daily activities.

However, civil engineers still lack standard procedures and protocols that, with the help of experts, laboratories and specialized institutions, could allow them to gather all the information that damaged buildings can reveal, in order to learn from them before the society's socio-economic activities demolish, cover up and eliminate the evidence. The transversality of this work aims at contributing to justify the need for these protocols.

In this earthquake, failures were caused by inconsistencies between the structure actually built and the design model, and they left valuable lessons that should not be missed. The lack of integrality and coherence among the strength, deformation and stability design of structural elements having the same function (columns and walls) was the failure cause. The infringement of design codes andknowledge established by our ancestors, by either ignorance, lack of supervision or malice, should not be the cause for a society to lose, in seconds, hundreds of lives and buildings, and suffer invaluable and permanent damage of its material, cultural, family and emotional heritage.

The origins of engineering go way back to the millenary history of humankind, although some people formally situate them in Babylon, given the existence of the Code of Hammurabi, one of the first legacies to pass sentences for engineering malpractice.

Consequently, the origin of engineering lies in an empirical regulatory epistemology, based on science and technology, but also on the inventiveness, creation and accumulated culture, where intuition, economy, society and ecology serve as sources to respond to a definition that requires learning from the lessons involved in their failures. Thus, it self-identifies as follows: "Engineering is a profession where the art of design, knowledge of science, mastering of technology and professional intuition gained from education, as well as the ancestors' and personal experiences, are wisely, creatively and economically applied to put the forces and resources of nature, humans and society at the service of humankind" (ABET, Gallegos H. 1995. Castañeda A., 2013).

6. References

ACI-318-95. (1995), Building Code Requirements for Structural Concrete. American Concrete Institute, USA.
ACI-318S-05. (2005), Requisitos de Reglamento para Concreto Estructural American Concrete Institute, USA..
ACI-318S-14. (2014), Requisitos de Reglamento para Concreto Estructural: American Concrete Institute.
Aguiar, R. (2010), Fallas frecuentes durante los terremotos. Revista ESPEctativa(4), 10-11.
Aznar A., García H., Ignacio J., Herrera J. O. and Cervera J. (2008), Conexión de forjados de hormigón a soportes metálicos. Paper presented at the IV CONGRESO ACHE. Congreso Internacional de Estructuras, Valencia. http://oa.upm.es/6018/1/articulo_ponencia_valencia.pdf
Bai Y., Nie J. and Cai C. (2008), New connection system for confined concrete columns and beams. II: Theoretical modeling. Journal of structural engineering, 134(12), 1800-1809.
Castañeda Á. (2013), Pedagogía, tecnologías digitales y Gestión de la información y el conocimiento en la enseñanza de la ingeniería (L. H. Félix Varela Ed. Primera ed.). La Habana. DOI: 10.13140/RG.2.1.1076.7529.
Código ecuatoriano de la construcción (2001), Requisitos generales de diseño.
Computers and Structures I. (2015), Computers & Structures Inc. Structural and Earthquake Engineering Software, ETABS 2015.
Gallegos H. (2011), El éxito de las fallas (C. d. I. d. Perú Ed.). Lima, Perú.
Gallegos H. (1995), Las fallas maestras de la ingeniería. Revista El Ingeniero Civil, No. 93 Año 13. Lima, Perú.
Guerra, M. (2015), Diseño sismo resistente de edificios de acero utilizando ETABS y la NEC 2015. (Primera ed.). Quito.
IG-EPN. (2016), Informe sísmico especial n. 18 - 2016. from http://www.igepn.edu.ec/servicios/noticias/1324-informe-sismico-especial-n-18-2016
Kuramoto H. and Nishiyama I. (2004), Seismic performance and stress transferring mechanism of through-column-type joints for composite reinforced concrete and steel frames. Journal of structural engineering, 130(2), 352-360.
Li W., Li Q.-n., Jiang W.-s. and Jiang L. (2011), Seismic performance of composite reinforced concrete and steel moment frame structures–state-of-the-art. Composites Part B: Engineering, 42(2), 190-206.
López O. and Espinoza L. (2016), Collapse of rmc high-school during the 1997 cariaco earthquake. Revista de la Facultad de Ingeniería, 31(1). doi: 10.21311/002.31.1.24
Meli R. (2001), Diseño Estructural (L. S. A. d. C.V. Ed. Primera ed.). México.
Mieles Y. and Castañeda E. (2016a), Análisis de vigas de hormigón armado con nudos híbridos. Paper presented at the XI Congresom de Ciencia & Tecnología UFA-ESPE, Quito.
Mieles Y. and Castañeda E. (2016b), Estudio de alteraciones en el comportamiento estructural de vigas de hormigón armado con nudos híbridos mediante el empleo de gráficos momento-curvatura. Revista Internacional de Ingeniería de Estructuras, 21,1, 45-59.
Mieles Y. and Castañeda E. (2016c), Reflexiones sobre daños observados en edificios de vigas con nudos híbridos y losas "steel-deck" ante el sismo del 16 de abril de 2016. Paper presented at the Proceedings of the "First Annual State-of-the-Art in Civil Engineering Structures and Materials", Quito.
Mieles Y. and Macías S. (2015), Análisis y diseño de una edificación de hormigón armado usando ETABS.
Milheiro J., Rodrigues H. and Arêde A. (2016), Evaluation of the contribution of masonry infill panels on the seismic behaviour of two existing reinforced concrete buildings. KSCE Journal of Civil Engineering, 20(4), 1365-1374. doi: 10.1007/s12205-015-0112-y
Norma Ecuatoriana de la Construcción (2015), Estructuras de Hormigón Armado.

Norma Ecuatoriana de la Construcción (2015), Riesgo sísmico, evaluación, rehabilitación de estructuras (2015)
Nie J., Bai Y. y Cai C. (2008), New connection system for confined concrete columns and beams. I: Experimental study. Journal of structural engineering, 134(12), 1787-1799.
Placencia P., Gallegos A. and Morales M. (2014), Análisis estructural de losas con luces de 6, 10, 12 metros utilizando dos sistemas constructivos. Escuela Politécnica Nacional, Ecuador
Priestley M. and Paulay T. (1992), Seismic design of reinforced concrete and masonry buildings. New York: John Wiley & Sons, Inc.
Pulido H. G. and De la Vara Salazar R. (2012), Análisis y diseño de experimentos (3ra Edición ed.): McGraw-Hill Interamericana.
Rasoul S., Bakhshayesh N. and Mehdi M. (2016), Moment-connection between continuous steel beams and reinforced concrete column under cyclic loading. Journal of Constructional Steel Research, 118, 105-119.
Schlaich J., Schäfer K. and Jennewein M. (1987), Toward a consistent design of structural concrete. PCI journal, 32(3), 74-150.
Soto J. (2012), Proyecto de conecciones de vigas de acero a muros de concreto en estructuras mixtas. Universidad Católica Andrés Bello.
Statgraphics Centurion X. (2009), Statpoint technologies.